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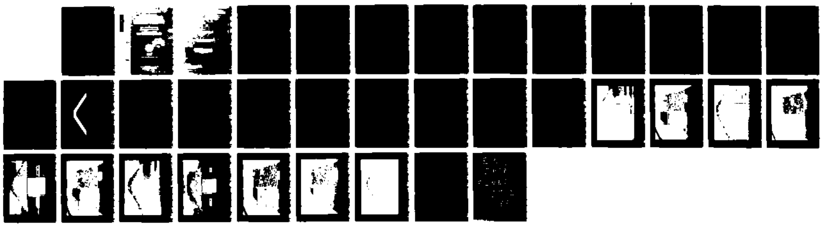

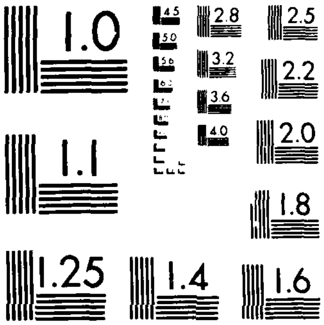
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Stability of Dolos Overlays for Rehabilitation of Tribar-Armored Rubble-Mound Breakwater and Jetty Trunks Subjected to Breaking Waves

12 PERSONAL AUTHOR(S)
Carver, Robert D.; Wright, Brenda J.

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19 ABSTRACT (Continue on reverse if necessary and identify by block number)

An experimental model investigation was conducted to obtain design guidance for dolos overlays used to rehabilitate tribar-armored rubble-mound breakwater and jetty trunks subject to breaking waves. It was concluded that:

a. The stability coefficient is independent of sea-side structure slope for slopes of 1V on 1.5H and 1V on 2H.

b. Stability showed some dependency on both d/L and H/d with minimum stability occurring at the lower values of d/L and higher values of H/d, i.e. longer wave periods in shallower water.

c. The minimum stability coefficient observed significantly exceeds that obtained for new construction.

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PREFACE

Authority to carry out this investigation was granted the US Army Engineer Waterways Experiment Station's (WES's) Coastal Engineering Research Center (CERC) by the Office, Chief of Engineers (OCE) under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program Work Unit 32325, "Use of Dissimilar Armor for Repair and Rehabilitation of Rubble-Mound Coastal Structures."

Tests of dolos overlays for existing tribar armor, which fulfill one milestone of this work unit, were conducted under the general direction of Mr. James E. Crews and Tony C. Liu, REMR Overview Committee, OCE; Mr. Jesse A. Pfeiffer, Jr., Directorate of Research and Development, OCE; members of the REMR Field Review Group; Mr. John H. Lockhart, Jr., Coastal Technical Monitor, OCE; Mr. William F. McCleese, REMR Program Manager, WES; and Mr. D. D. Davidson, REMR Coastal Program Area Leader, CERC.

The study was conducted by personnel of CERC under the general direction of Dr. James R. Houston, Chief, CERC, and Mr. Charles C. Calhoun, Jr., Assistant Chief, CERC; and under direct supervision of Mr. C. E. Chatham, Chief, Wave Dynamics Division, and Mr. D. D. Davidson, Chief, Wave Research Branch. Tests were planned by Mr. Robert D. Carver, Principal Investigator, and Ms. Brenda J. Wright, Civil Engineering Technician. The model was operated by Ms. Wright, under the supervision of Mr. Carver. This report was prepared by Mr. Carver and Ms. Wright and edited by Ms. Shirley A. J. Hanshaw, Information Products Division, Information Technology Laboratory, WES.

Director of WES during report publication was COL Dwayne G. Lee, CE. Technical Director was Dr. Robert W. Whalin.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	25.4	millimetres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres

STABILITY OF DOLOS OVERLAYS FOR REHABILITATION OF TRIBAR-ARMORED RUBBLE-MOUND BREAKWATER AND JETTY TRUNKS SUBJECTED TO BREAKING WAVES

PART I: INTRODUCTION

Background

1. The experimental investigation described herein constitutes a portion of a research effort to provide engineering data for the effective and economical rehabilitation of rubble-mound breakwaters and jetties. In this study, a rubble-mound breakwater or jetty is defined as a protective structure constructed with a core of quarry-run stone, sand, or slag and protected from wave action by one or more stone underlayers and a cover layer composed of selected quarrystone or specially shaped concrete armor units.

2. Previous investigations, under Work Unit 31269, "Stability of Breakwaters," have yielded a significant quantity of design information for new construction using quarrystone (Hudson 1958 and Carver 1980 and 1983), tetrapods, quadripods, tribars, modified cubes, hexapods, and modified tetrahedrons (Jackson 1968), dolosse (Carver and Davidson 1977 and Carver 1983), and toskane (Carver 1978). Rehabilitation projects on several of the Corps' rubble-mound structures have revealed a total lack of design guidance or even information concerning the interfacing and stability response of armor units that are of dissimilar type and/or size. In the past, selection of new armor type, method of interfacing, and procedures for preparation of the existing section have been based on engineering judgment or, in more recent times, on site-specific model studies. The engineering judgment process can be expensive since experience is limited and there is not usually a solid basis for it. This process can lead to recurring failures that cost millions of dollars without a real solution being developed for the long-term problem. Site-specific model studies have provided good singular solutions, but site-specific data usually fail to meet the requirements of other projects (Carver, in preparation). It is anticipated that the problem will become more acute in future years as rehabilitation of major breakwaters and jetties becomes necessary to extend their project life or to meet greater design demands.

Approach

3. Model breakwaters and armor units are being used to experimentally investigate the stability response of various armor combinations for selected structure geometries and wave conditions. It would be an extremely extensive task to comprehensively investigate all different types of existing armor units; therefore, this research effort will address only the three types (stone, dolos, and tribars) of armor most commonly used in the Corps. Selection of these armor types should give test results the widest range of applicability possible. Tests will be conducted with breaking wave conditions on no-damage, no-overtopping breakwater trunk and head sections using sea-side slopes of 1V:1.5H and 1V:2H. Test results for dolos and tribar overlays of existing stone armor and dolos overlays of existing dolos have been reported (Carver and Wright 1987a and 1987b).

Purpose of Study

4. The purpose of the present investigation was to obtain design guidance for dolos overlays used to rehabilitate tribar-armored rubble-mound breakwater and jetty trunks subjected to breaking waves. More specifically, it was desired to determine the minimum weight of individual armor units (with given specific weights) required for stability as a function of:

- a. Sea-side slope of the structure.
- b. Wave period.
- c. Wave height.
- d. Water depth.

PART II: TESTS

Stability Scale Effects

5. If the absolute sizes of experimental breakwater materials and wave dimensions become too small, flow around the armor units enters the laminar regime; and the induced drag forces become a direct function of the Reynolds number. Under these circumstances prototype phenomena are not properly simulated, and stability scale effects are induced. Hudson (1975) presents a detailed discussion of the design requirements necessary to ensure the preclusion of stability scale effects in small-scale breakwater tests and concludes that scale effects will be negligible if the Reynolds stability number (R_N)*

$$R_N = \frac{g^{1/2} H^{1/2} \ell_a}{\nu}$$

where

g = acceleration due to gravity, ft/sec^2

H = wave height, ft

ℓ_a = characteristic length of armor unit, ft

ν = kinematic viscosity

is equal to or greater than 3×10^4. For all tests reported herein, the sizes of experimental armor and wave dimensions were selected such that scale effects were insignificant (i.e., R_N was greater than 3×10^4).

Test Procedures

Method of constructing test sections

6. All experimental breakwater sections were constructed to reproduce as closely as possible results of the usual methods of constructing full-scale breakwaters. The core material was dampened as it was dumped by bucket or shovel into the flume and was compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype

* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix A).

structure. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure adequate compaction of the material. The underlayer stone then was added by shovel and smoothed to grade by hand or with trowels. No excessive pressure or compaction was applied during placement of the underlayer stone. Armor units used in the cover layers were placed in a random manner corresponding to work performed by a general coastal contractor, i.e., they were individually placed but were laid down without special orientation or fitting. After each test series the armor units were removed from the breakwater, all of the underlayer stones were replaced to the grade of the original test section, and the armor was replaced.

Selection of critically breaking waves

7. For a given wave period and water depth, the most detrimental breaking wave (i.e. the most damaging wave) was determined by increasing the stroke adjustment on the wave generator in small increments and observing which wave produced the most severe breaking wave condition on the experimental structures. Wave heights of lower amplitude did not form the critical breaking wave, and wave heights of larger amplitude would break seaward of the test structures and dissipate their energy so that they were less damaging than the critically tuned wave.

8. A typical stability test series consisted of subjecting the test sections to attack by waves of given heights and periods until all damage had abated or the structures failed. Test sections were subjected to wave attack in approximately 30-sec intervals between which the wave generator was stopped and the waves allowed to decay to zero height. This procedure was necessary to prevent the structures from being subjected to an undefined wave system created by reflections from the experimental breakwater and wave generator. Newly built test sections were subjected to a short duration (five or six 30-sec intervals) of shakedown using a wave equal in height to about one-half of the design wave. This procedure provided a means of allowing consolidation and armor unit seating simulating that which would normally occur during prototype construction.

Method of determining damage

9. To evaluate and compare breakwater stability test results, it is necessary to quantify the changes that have taken place in a given structure during attack by waves of specified characteristics. The US Army Engineer Waterways Experiment Station (WES) developed a method of measuring the

percentage of damage incurred by a test section during the early 1950's. This method has proven satisfactory and was used as a means for analyzing and comparing the stability tests delineated herein.

10. The WES damage-measurement technique requires that the cross-sectional area occupied by armor units be determined for each stability test section. Armor unit area is computed from elevations (soundings) taken at closely spaced grid-point locations before the armor is placed on the underlayer, after the armor has been placed but before the section has been subjected to wave attack, and finally after wave attack. Elevations are obtained with a sounding rod equipped with a circular spirit level for plumbing, a scale graduated in thousandths of a foot, and a ball-and-socket foot for adjustment to the irregular surface of the breakwater slope. The diameter in inches of the circular foot of the sounding rod was related to the size of the material being sounded by the following equation:

$$\text{Diam} = C\left(\frac{W_a}{\gamma_a}\right)^{1/3}$$

where

C = coefficient

W_a = weight of an armor unit, lb

γ_a = specific weight of armor unit, pcf

$C = 6.8$ for tribars and stone and 13.7 for dolosse. A series of sounding tests in which both the weight of the armor and the diameter of the sounding foot were varied indicated that the above relation would give a measured thickness which visually appeared to represent an acceptable two-layer thickness.

11. Sounding data for each test section were obtained as follows: after the underlayer was in place, soundings were taken on the slopes of the structure along rows beginning at and parallel to the longitudinal center line of the structure and extending in 0.25-ft* horizontal increments until the edge of the armor was reached. On each parallel row, sounding points, spaced

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* A table of factors for converting non-SI units of measurement to SI metric units is presented on page 3.

at 0.25-ft increments, were measured. The 0.5 ft of structure next to each wall was not considered because of the possibility of discontinuity effects between armor units and the flume walls. Soundings were taken at the same points once the armor was in place and again after the structure had been subjected to wave attack.

12. Sounding data from each stability test were reduced in the following manner. The individual sounding points obtained on each parallel row were averaged to yield an average elevation at the bottom of the armor layer before the armor was placed and then at the top of the armor layer before and after testing. From these values, the cross-sectional armor area before testing and the area from which armor units were displaced (either downslope or off the section) were calculated. Damage then was determined from the following relation:

$$\text{Percent damage} = \frac{A_2}{A_1} (100)$$

where

A_1 = area before testing, ft^2

A_2 = area from which armor units have been displaced, ft^2

The percentage given by the WES sounding technique is, therefore, a measurement of an end area which converts to an average volume of armor material that has been moved from its original location (either downslope or off structure).

Test Equipment

13. All tests were conducted in a 5-ft-wide, 4-ft-deep, 119-ft-long concrete wave flume with test sections installed about 90 ft from a vertical displacement wave generator. A thin divider was installed in the center of the test section area, thus yielding two 2.5-ft-wide sections. The first 10-ft length of flume bottom, immediately seaward of the test sections, was molded on a 1V-on-10H slope, while the remaining 80-ft length was flat. The generator is capable of producing sinusodial waves of various periods and heights. For all tests, waves of the required characteristics were generated by varying the frequency and amplitude of the plunger motion. Changes in water surface elevation as a function of time (wave heights) were measured by

electrical wave height gages in the vicinity of where the toe of the test sections was to be placed (without the structure in place) and recorded on chart paper by an electrically operated oscillograph. The electrical output of the wave gages was directly proportional to their submergence depth.

Selection of Test Conditions

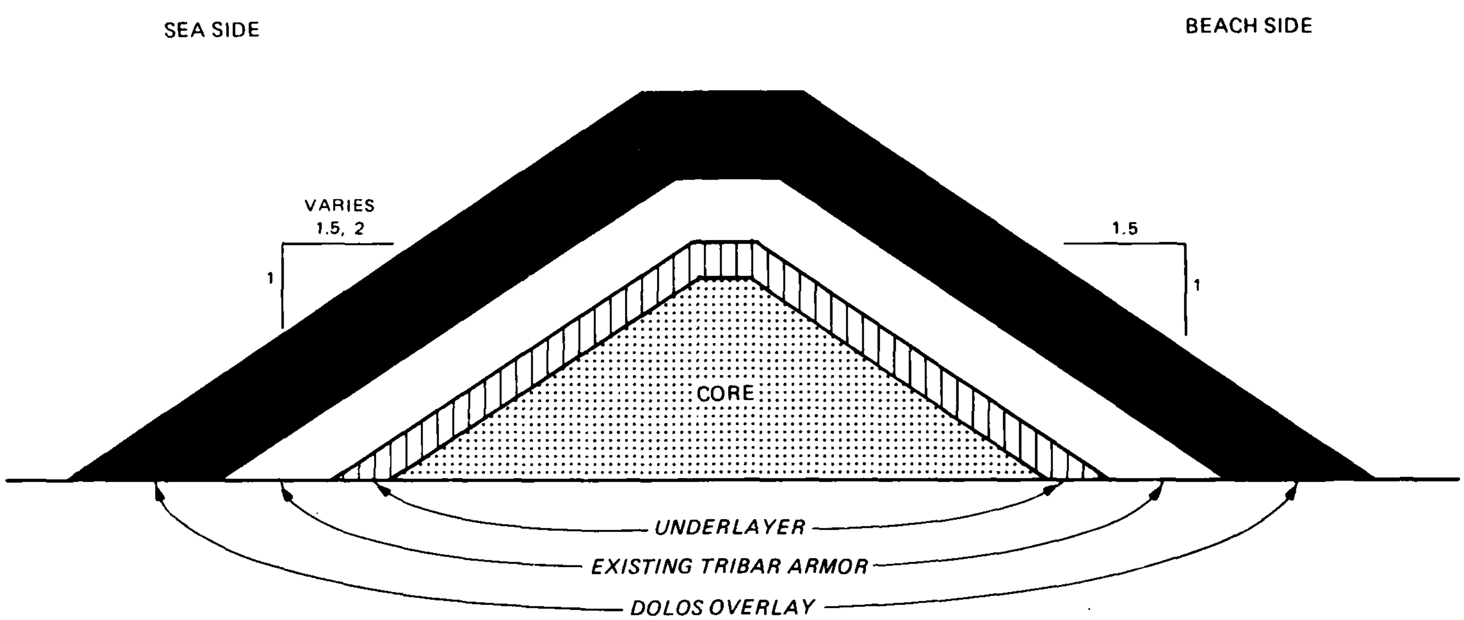
14. Breaking wave tests were conducted using dolos overlays. A review of past site-specific stability projects and hydrographic data showed that typical prototype sea-bottom slopes could range from almost flat to as steep as 1V on 10H. Realizing that wave deformation and severity of breaking action increases as bottom slope increases, and since time constraints would allow testing of only one slope, it was decided to use a 1V-on-10H slope, thus ensuring severe depth-limited breaking wave action (plunging breakers). When breaking directly on the structure, this type of wave normally causes the most damage to rubble-mound structures.

15. By nondimensionalizing design conditions from site-specific projects, it was found that a relative depth (d/L) range of 0.4 to 0.14 should include most prototype conditions encountered in breaking wave stability designs. A review of capabilities of the available flume and wave generator showed that this range of d/L values could be achieved for a reasonable range of testing depths.

16. The wave flume was calibrated for depths from 0.40 to 1.00 ft in 0.05-ft increments at d/L values of 0.04, 0.06, 0.08, 0.10, 0.12, and 0.14. This range of depths, and consequently breaking wave heights, proved to be compatible with the selected armor weights and sea-side breakwater slopes.

17. All stability tests were conducted on sections of the type shown in Figure 1 and Photos 1-4. Sea-side slopes of 1V on 1.5H and 1V on 2H were investigated, while the beach-side slope was held constant at 1V on 1.5H. Heights of the simulated existing structures (prior to placement of the dolos overlays) varied from 1.0 to 1.2 ft. The height necessary to prevent wave overtopping of the existing structure was determined from the slopes, water depths, and wave heights investigated in determining stability coefficients for the dissimilar armor overlays.

18. It was assumed that the overlaying dolos armor could be slightly to significantly smaller than the existing tribars. A review of existing model



NOTE: TEST SECTIONS FRONTED BY A 1V-ON-10H BOTTOM SLOPE.

ARMOR TYPE	WEIGHT, LB
EXISTING TRIBARS	0.627
DOLOS OVERLAY	0.442
DOLOS OVERLAY	0.589

Figure 1. Typical breakwater cross section

materials was made in concert with this assumption, and 0.627-lb tribars were selected to simulate existing conditions. Tribars were randomly placed in two layers. Overlaying dolos weights of 0.442 and 0.589 lb were used.

PART III: TEST RESULTS

19. Various combinations of wave height and period and water depth were investigated for the selected armor weights and structure slopes. Some of these conditions proved to be too severe, i.e., they produced excessive damage as measured by the sounding method. Conversely, some conditions proved to be conservative. Results of those tests which yielded stable design conditions are summarized in Table 1. Presented therein are experimentally determined design wave heights and calculated stability coefficients K_D's as functions of relative depth d/L and relative wave heights H/d. The stability coefficient K_D is determined from the Hudson formula, i.e.,

$$W_a = \frac{\gamma_a H^3}{K_D \left(S_a - 1\right)^3 \cot \alpha}$$

where

K_D = stability coefficient

S_a = specific gravity of armor unit

α = reciprocal of breakwater slope

Armor units were placed randomly in two layers, and the number of armor units per given surface area was equal to that presently recommended for new construction in EM 1110-2-2904 (Headquarters, Department of the Army 1986). Photos 5-11 show typical after-testing conditions of the structures.

20. Figures 2 and 3 present K_D as a function of d/L, H/d, and sea-side structure slope. These data show the stability coefficient to be independent of sea-side structure slope; however, a slight dependency on both d/L and H/d is observed with minimum stability occurring at the lower values of d/L and higher values of H/d, i.e. longer wave periods in shallower water.

21. The minimum stability coefficient (20) observed in the present investigation is very significant. Previous tests of dolos overlays for existing stone armor (Carver and Wright 1988a) and existing dolosse (Carver and Wright 1988b) yielded minimum stability coefficients of 12 and 15. Thus, the obtained value of 20 significantly exceeds that observed for other dissimilar armor combinations and present recommendations for new construction (K_D = 15).

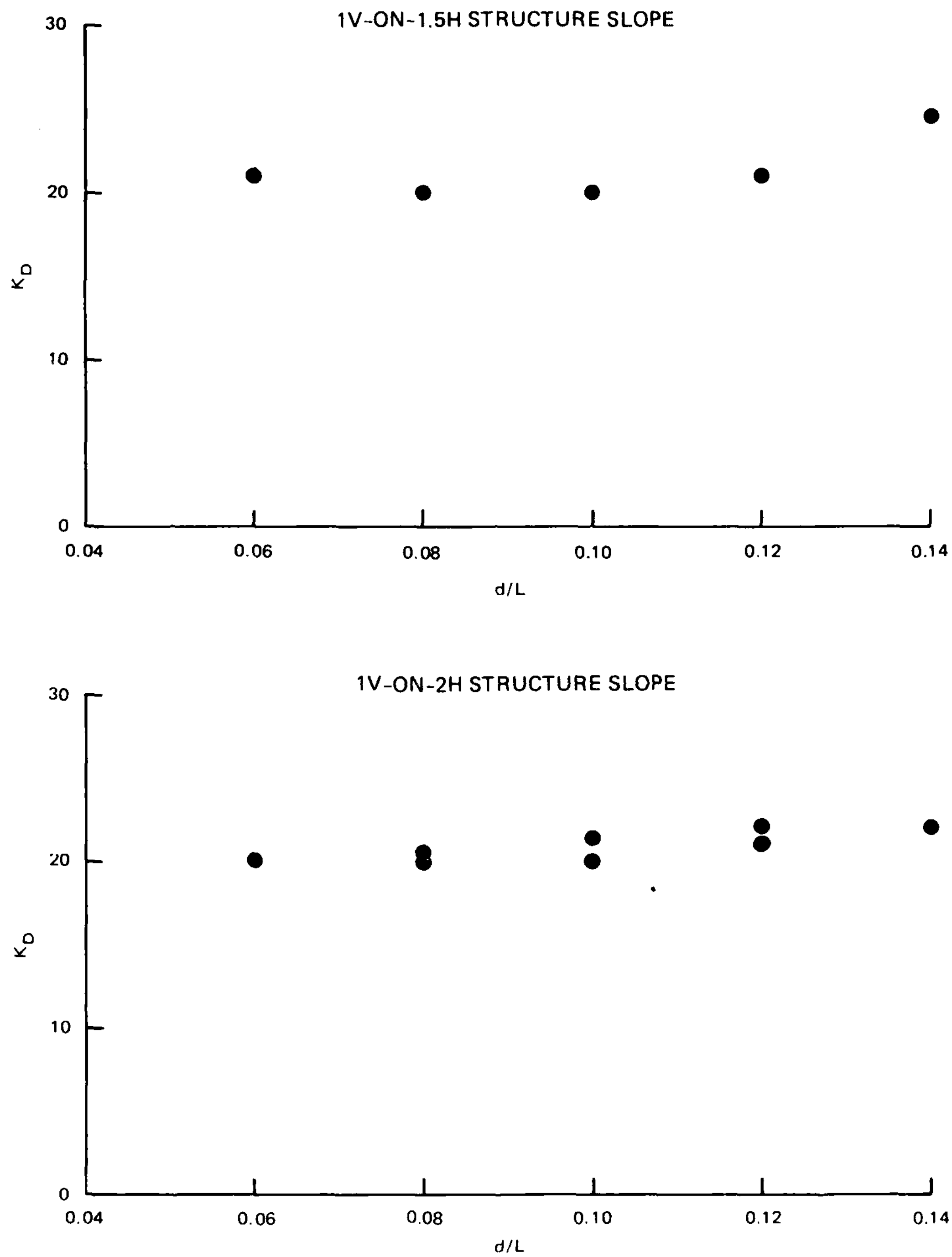


Figure 2. Stability coefficient (K_D) versus relative depth (d/L)

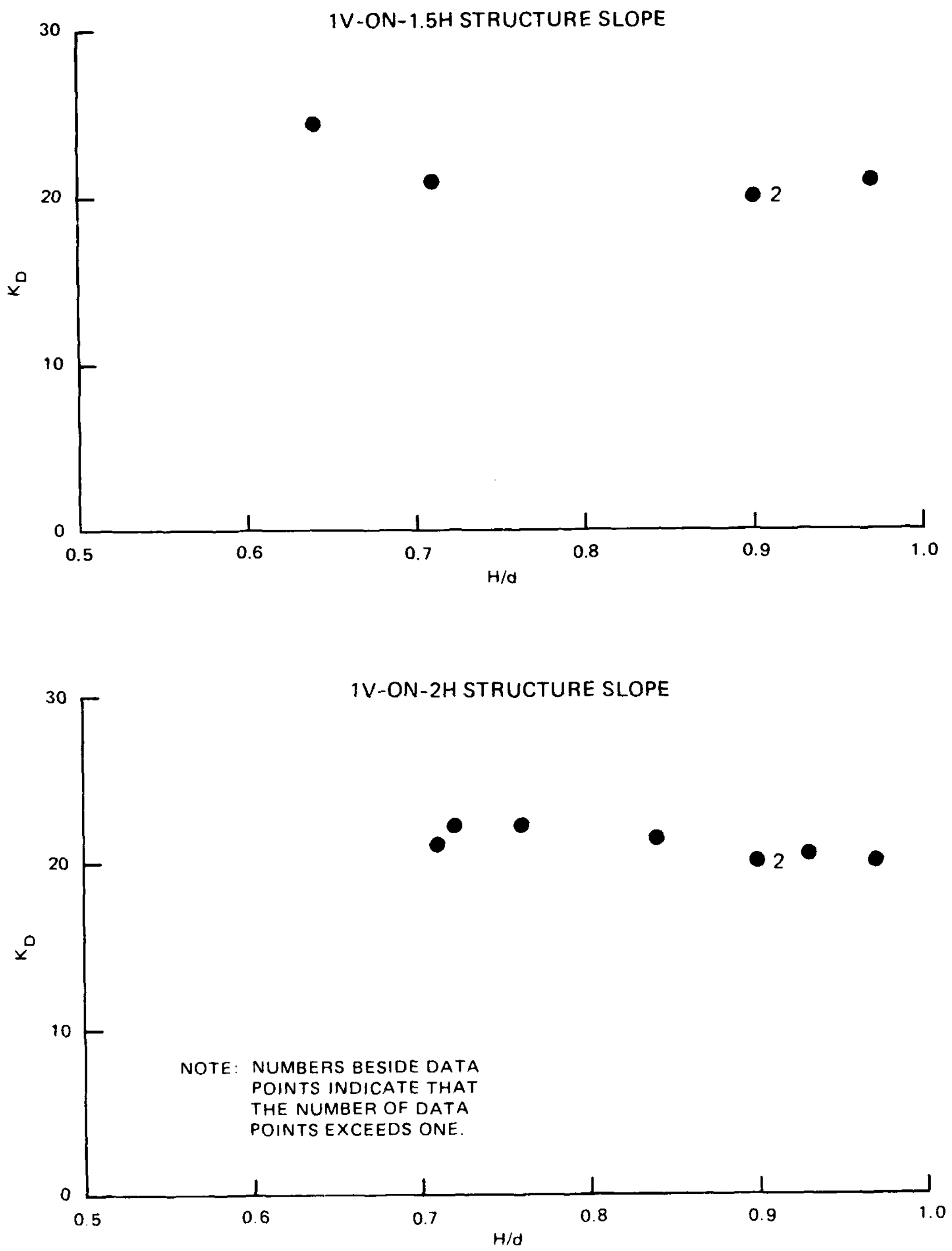


Figure 3. Stability coefficient (K_D) versus relative wave height (H/d)

Therefore, due to superior stability, a tribar dolos combination might be considered for new construction.

PART IV: CONCLUSIONS

22. Based on tests and results described herein in which dolos armor is used to overlay existing tribars on breakwater trunks subjected to breaking waves with a direction of approach of 90 deg, it is concluded that:

<u>a</u>. The stability coefficient is independent of sea-side structure slope for slopes of 1V on 1.5H and 1V on 2H.

<u>b</u>. Stability showed some dependency on both d/L and H/d with minimum stability occurring at the lower values of d/L and higher values of H/d, i.e. longer wave periods in shallower water.

<u>c</u>. The minimum stability coefficient observed significantly exceeds that obtained for new construction.

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Table 1

Values of H , d/L , H/d , and K_D for Dolos Overlays of Existing Tribar Armor Subjected to Breaking Waves

W_a, lb	d, ft	T, sec	H, ft	d/L	H/d	K_D
			1V-on-1.5H Structure Slope			
0.442	0.60	2.32	0.58	0.06	0.97	21.0
0.442	0.95	1.37	0.61	0.14	0.64	24.5
0.589	0.70	1.57	0.63	0.10	0.90	19.9
0.589	0.70	1.92	0.63	0.08	0.90	19.9
0.589	0.90	1.52	0.64	0.12	0.71	20.8
			1V-on-2H Structure Slope			
0.442	0.65	2.42	0.63	0.06	0.97	20.2
0.442	0.70	1.57	0.63	0.10	0.90	20.2
0.442	0.70	1.92	0.63	0.08	0.90	20.2
0.442	0.90	1.52	0.64	0.12	0.71	21.2
0.589	0.75	1.99	0.70	0.08	0.93	20.5
0.589	0.85	1.73	0.71	0.10	0.84	21.4
0.589	0.95	1.56	0.72	0.12	0.76	22.3
0.589	1.00	1.40	0.72	0.14	0.72	22.3

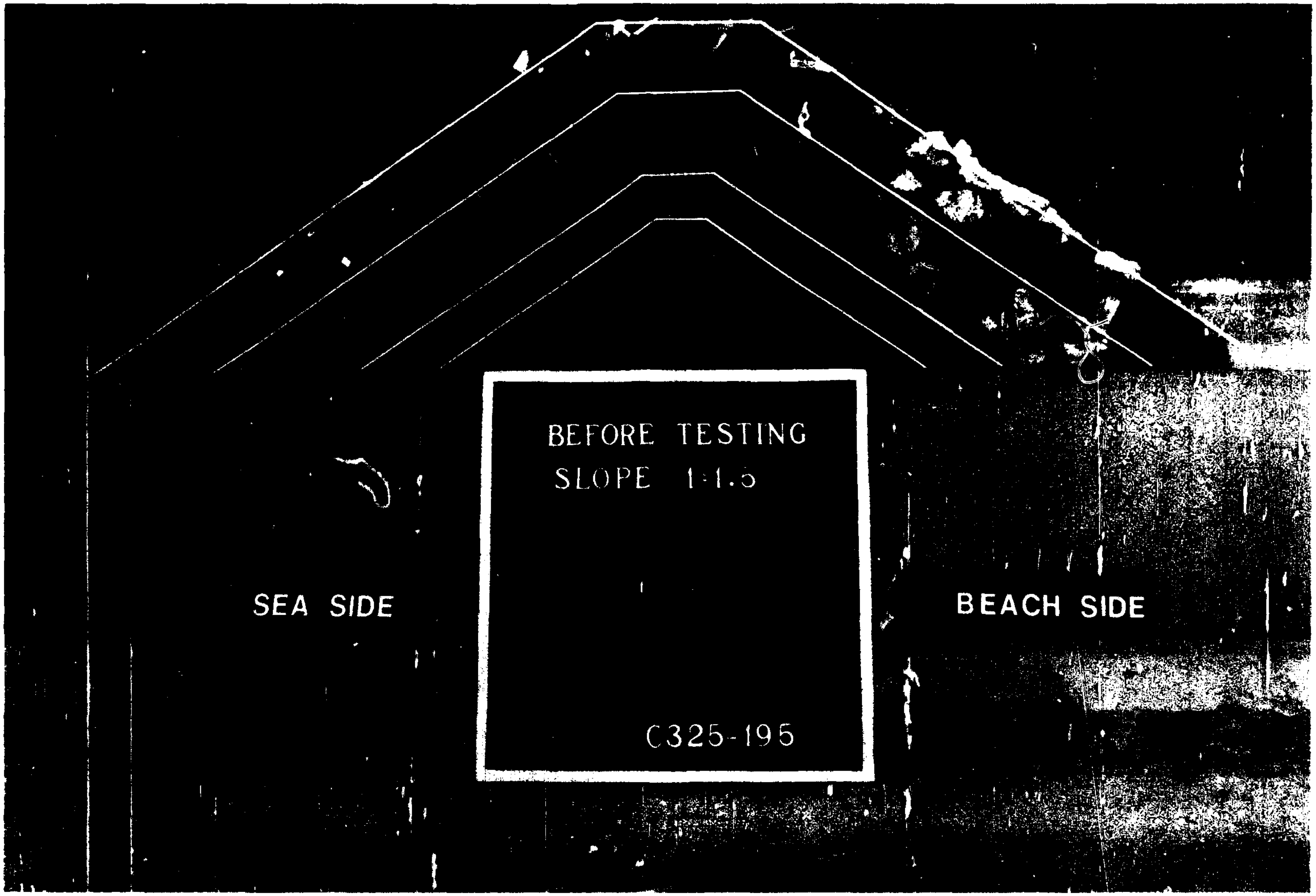

Photo 1. End view of a typical test section before wave attack at a 1V-on-1.5H sea-side structure slope; $W_a = 0.442$ lb

Photo 2. Sea-side view of a typical test section before wave attack at a 1V-on-1.5H sea-side structure slope; W_a = 0.442 lb

Photo 3. End view of a typical test section before wave attack at a 1V-on-2H sea-side structure slope; W_a = 0.589 lb

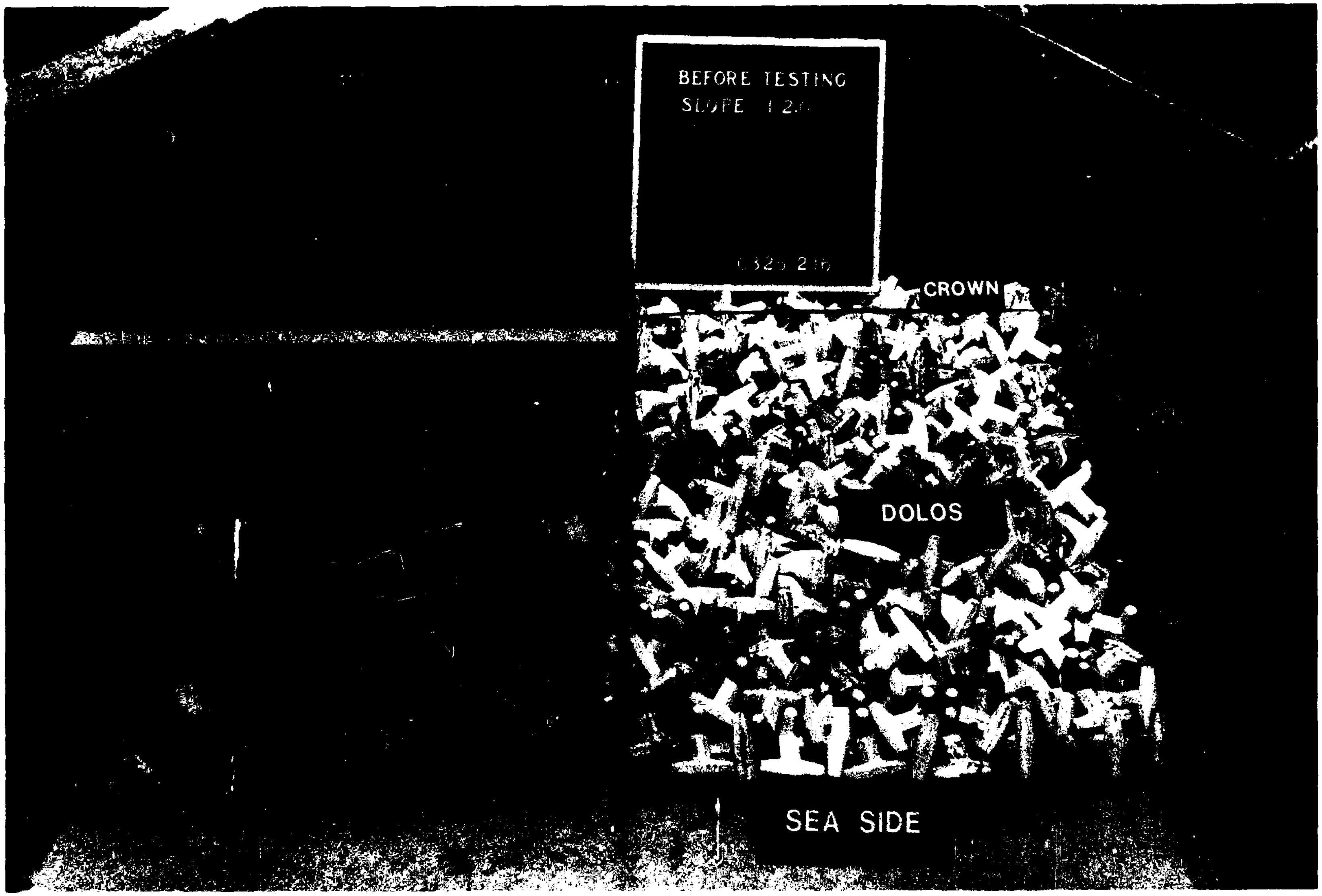

Photo 4. Sea-side view of a typical test section before wave attack at a 1V-on-2H sea-side structure slope; W_a = 0.589 lb

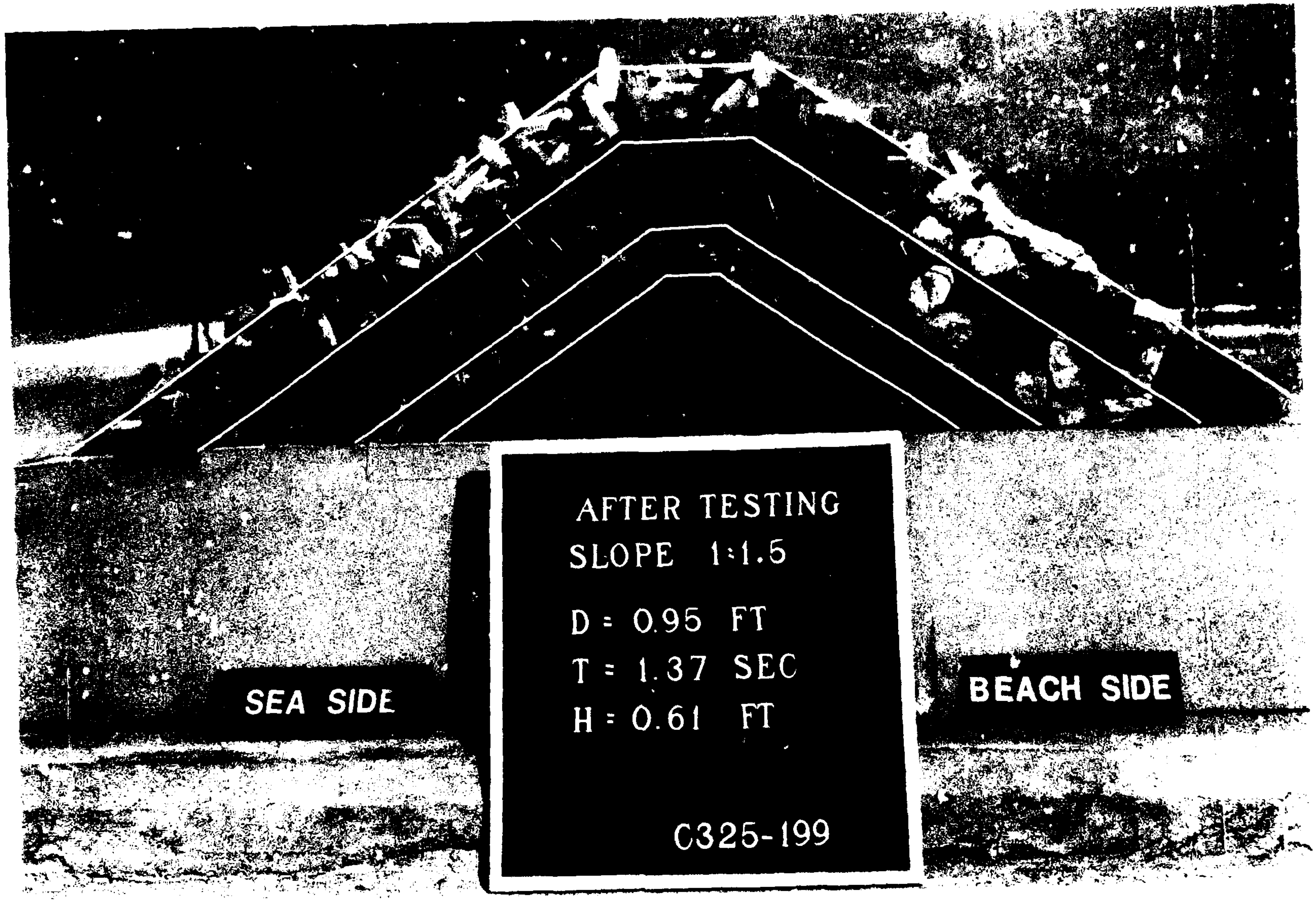

Photo 5. End view after attack of 1.37-sec, 0.61-ft waves; d = 0.95 ft; W_a = 0.442 lb; 1V-on-1.5H structure slope

Photo 6. Sea-side view after attack of 1.57-sec, 0.63-ft waves; d = 0.70 ft; W_a = 0.589 lb; 1V-on-1.5H structure slope

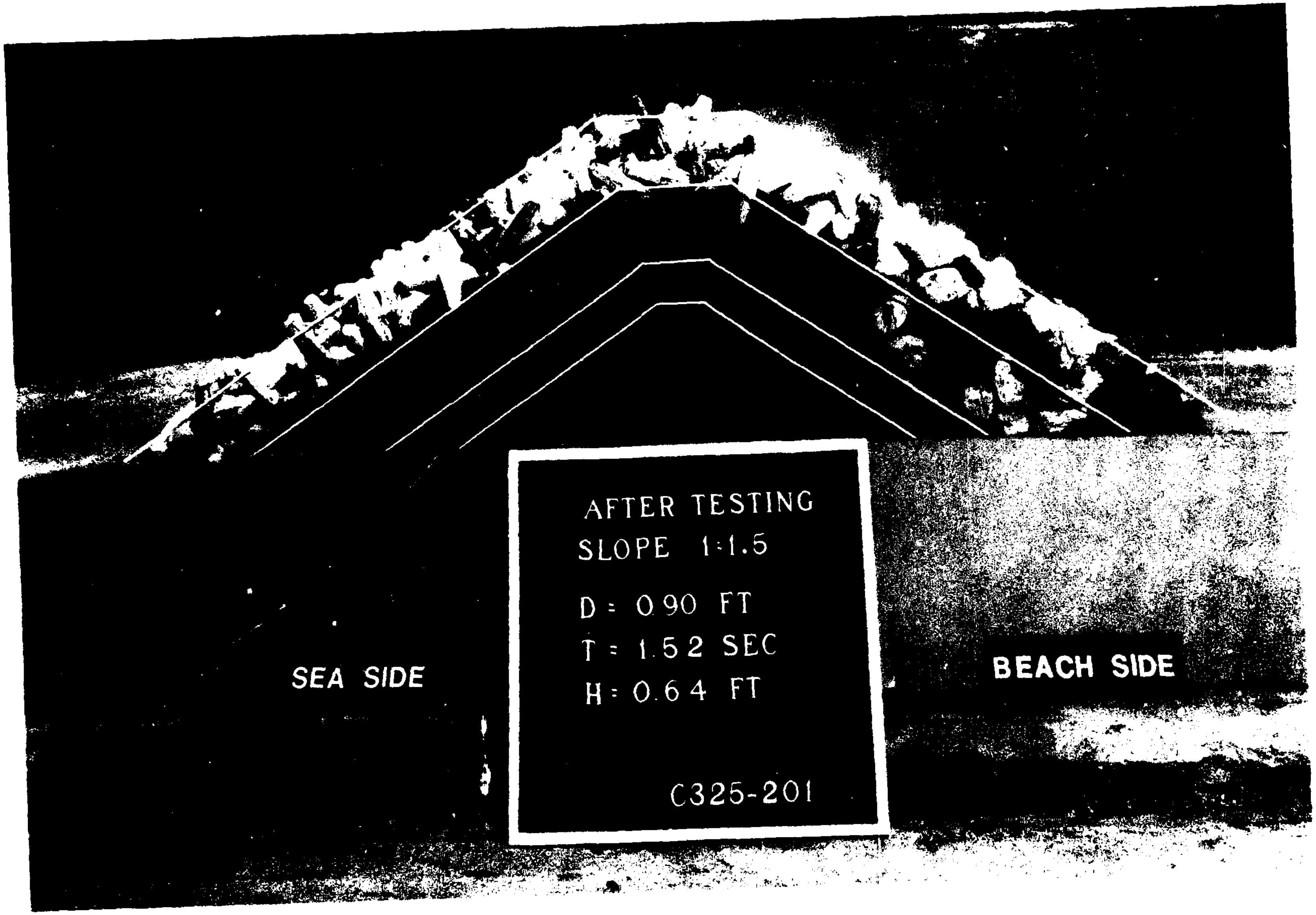

Photo 7. End view after attack of 1.52-sec, 0.64-ft waves; d = 0.90 ft; W_a = 0.589 lb; 1V-on-1.5H structure slope

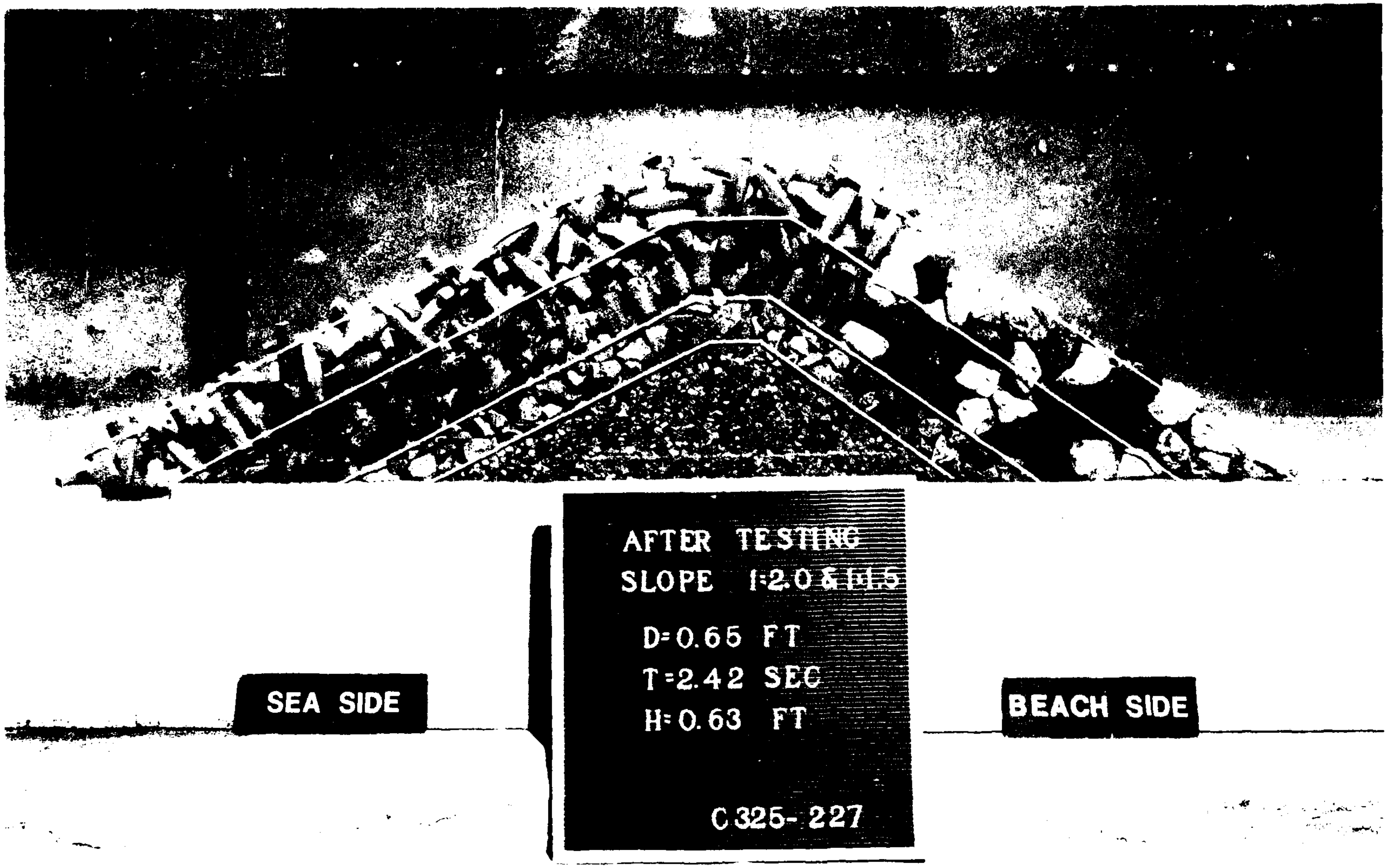

Photo 8. End view after attack of 2.42-sec, 0.63-ft waves; d = 0.65 ft; W_a = 0.442 lb; 1V-on-2H structure slope

Photo 9. Sea-side view after attack of 1.52-sec, 0.64-ft waves; $d = 0.90$ ft; $W_a = 0.442$ lb; 1V-on-2H structure slope

Photo 10. Sea-side view after attack of 1.73-sec, 0.71-ft waves; $d = 0.85$ ft; $W_a = 0.589$ lb; 1V-on-2H structure slope

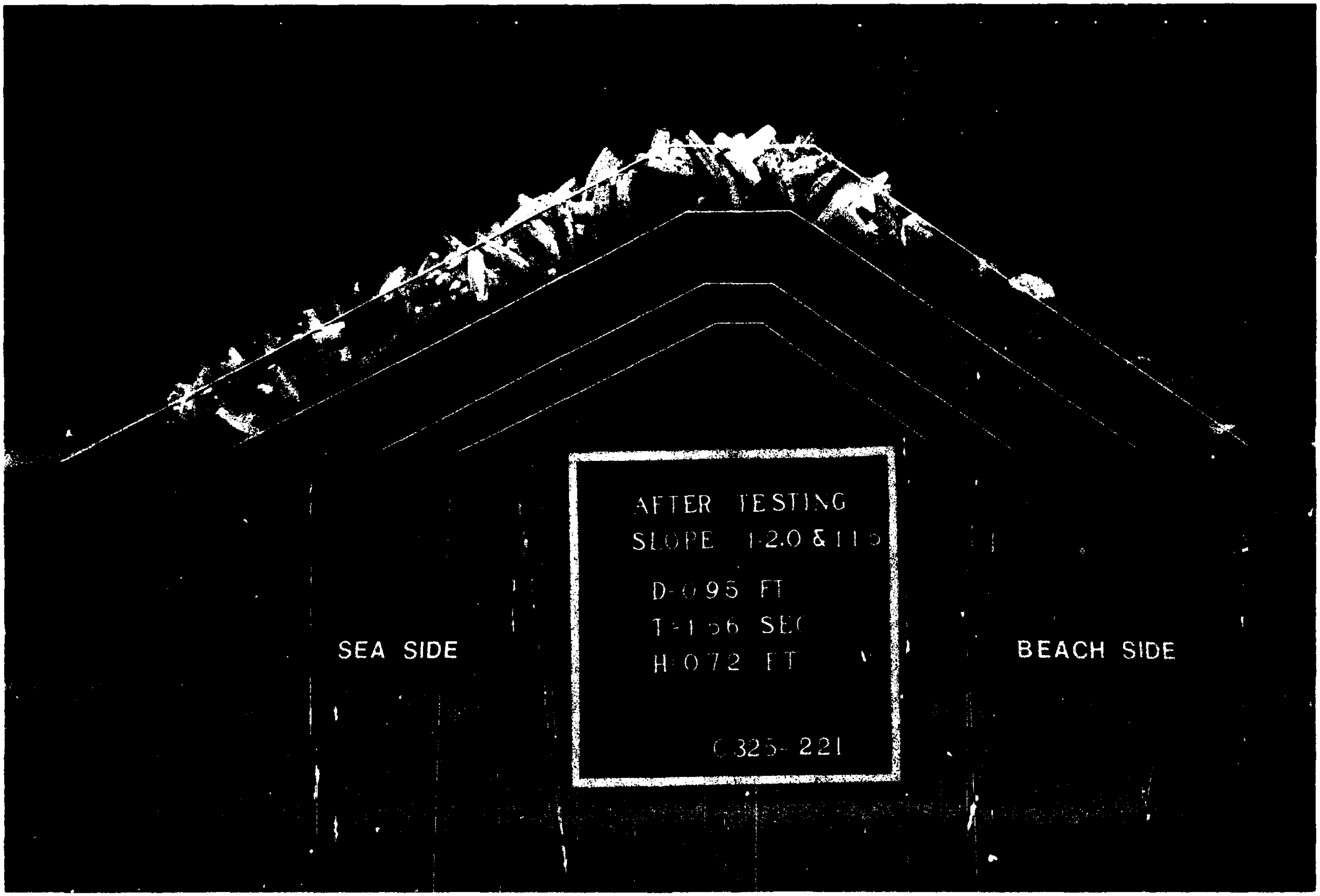

Photo 11. End view after attack of 1.56-sec, 0.72-ft waves; $d = 0.95$ ft; $W_a = 0.589$ lb; 1V-on-2H structure slope

APPENDIX A: NOTATION

A_1 and A_2	Surface area, ft^2
C	Coefficient
d	Water depth, ft
d/L	Relative depth
g	Acceleration due to gravity, ft/sec^2
H	Wave height, ft
H/d	Relative wave height
K_D	Stability coefficient
ℓ_a	Characteristic length of armor unit, ft
R_N	Reynolds stability number = $g^{1/2}H^{1/2}\ell_a/\nu$
T	Wave period sec, time
W_a	Weight of an armor unit, lb
cot α	Reciprocal of breakwater slope
γ_a	Specific weight of an armor unit, pcf
ν	Kinematic viscosity